Geosynthetic Reinforced Soil Integrated Bridge System

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ABSTRACT

Presented is the concept of a Geosynthetic Reinforced Soil Integrated Bridge System. This simple method of bridge support blends the superstructure with the substructure and the approach way. Different from the "Integral Abutment," the concept was developed by the Federal Highway Administration (FHWA) to meet the demand for new bridges as part of the "Bridge of the Future Program." This FHWA program was initiated to develop technologies to build more efficient, durable 70 to 90 foot single-span bridges.

This method of bridge support has been successfully used on many bridges in Defiance, Ohio. The method of construction utilizes geosynthetic–reinforced soil (GRS) to support prestressed concrete box beams directly on GRS abutments, without the use of a deep foundation or spread footing to support the superstructure. The GRS abutments are supported on a Reinforced Soil Foundation (RSF) over the subsoil. The bridge has no cast-in-place concrete. The bridge also does not have an approach slab or construction joint at the bridge to road interface. The roadway and the bridge are designed to settle together to provide a bump free, smooth ride over the bridge. The bridge system is expected to be easier to maintain because it has less parts.

It has been demonstrated that this GRS integrated bridge system can be built in less than two weeks for about 25% less cost than a bridge supported on pile-capped abutments with 2:1 slopes. Other advantages are that the construction of this bridge system is less dependent on the weather, and can be easily modified to account for unexpected field conditions and other unexpected design and construction issues.

The paper also addresses some of the challenges faced by FHWA in the implementation of new bridge technologies.

INTRODUCTION

The need to build bridges faster, better and less expensive is essential in maintaining the transportation infrastructure in the United States. Current practice cannot meet the combined demand for the construction of new bridges and the replacement of existing bridges. The current methods of bridge construction is time consuming and costly in terms of design, labor, materials, and traffic delays. These factors, in combination with the reduction of construction budgets of many agencies have created a situation that is insufficient to meet the demand for new bridges. The result is an unsustainable backlog of bridge replacement projects that will increase as the national highway system ages. To meet the demand for these new bridges, the Federal Highway Administration (FHWA) initiated the Bridge of the Future (BOF) program to develop new technologies to build better, more efficient bridge systems.

There are approximately 500,000 bridges in the national inventory. The majority, about 80% or 400,000, are single span bridges 70-90 feet in length. For this reason, the BOF program is geared toward these basic, "bread and butter" bridges that comprise the vast majority of the inventory. The objective of the program is to develop cost-effective design and efficient construction techniques for 70-90 foot bridge systems that deliver improved durability, maintenance, inspection accessibility and long-term performance.

Despite their intended service life of 75 years, many of the bridges built within the last 30 to 40 years are in need of replacement. These bridges are deficient because they fail to meet current load-rating criteria, inhibit the flow of traffic, bear on unknown foundations in scour susceptible soils, or are just deteriorating due to the effects of corrosion, fatigue and age.

While there are many methods to improve the efficient design of simple bridges, the focus of this initiative is to simplify the design and construction of the foundation for the simple bridge. Today, the majority of all bridges are built on deep foundations. The installation of a deep foundation contributes significantly to both the time and expense of the bridge. Where appropriate, the deep foundation can be efficiently replaced with a shallow foundation system. This simplified method of design may lead to the elimination of the conventional bridge seat, stem wall and approach slab on future simple bridges.

GEOSYNTHETIC REINFORCED SOIL TECHNOLOGY

Geosynthetic reinforced soil technology (GRS) has been used in the United States for more than three decades; it was first used in the 1970's by the U.S. Forest Service (USFS) to support logging roads in steep mountain terrain. Many of these GRS walls were built with plain non-woven geotextiles. Some of the walls used sawdust instead of compacted soil as backfill. Shortly after the construction of these USFS walls, the Colorado Department of Transportation (CDOT) began investigating the technology as part of the Interstate 70 Glenwood Canyon expansion project. Later in the 1990s, the Colorado Transportation Institute (CTI) produced several instructional videos on Generic Mechanically Stabilized Backfill Systems along with a manual to design and construct low cost retaining walls (T.H. Wu, 1994 [1]).

In the mid 1990s, the FHWA then partnered with CDOT to further improve the technology. During the past decade, the technology has been refined and successfully used to construct retaining walls, slopes, embankments, bridge abutments, culverts, rock fall barriers and foundations. To develop the technology for bridge support, the FHWA tested several full-scale GRS experiments at its Turner-Fairbank Highway Research Center in McLean, Virginia (M. Adams, 1997[2], M. Koklanaris, 2000[3]). CDOT also constructed and performed a long-term load test on a full-scale GRS pier and abutment combination at the Colorado DOT Havana maintenance facility (K.Ketchart and

J.T.H.Wu, 1997[4]). The research has also been applied to economically build many bridges in the private sector; these bridges are performing well.

Today, GRS is often mistakenly lumped with Mechanically Stabilized Earth (MSE) systems. An effort is underway to differentiate these two wall systems (CDOT-DTD-R-2001-16[5]). The main differences between GRS and MSE are the design methodologies: a GRS mass consists of closely spaced alternating layers of geosynthetic reinforcement and a compacted granular fill, producing a composite mass with different material properties from that of soil (NCHRP Report 556 [6]).

GRS is internally supported and can be designed without many of the factors governing the requirement of MSE systems. The behavioral differences are related to: 1) lateral earth pressure; 2) relationship between spacing and strength of reinforcement; 3) failure mechanism; and 4) safety factors. A GRS wall can be built without the MSE requirement for special block connections, reinforcement pullout, reinforcement creep reduction factors, specified wall embedment, specified wall base to height ratios, and lateral earth pressure at the face.

A GRS mass can be considered unique; a composite built with a particular compacted fill and reinforcement schedule producing a material with predicable properties. Research has been completed to develop test procedures used to evaluate the performance of a particular GRS mass (FHWA-RD-01-018 [7]).

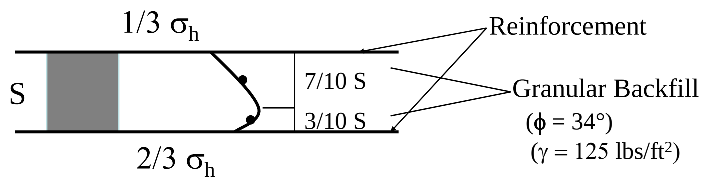
GRS walls and abutments can be built with readily available materials, using common construction equipment, and without highly skilled labor. These structures are also extremely durable and have been reported to perform very well in earthquakes, if constructed properly and with closely spaced reinforcement.

There are two basic rules to assure acceptable performance of a GRS mass: 1) good compaction with quality granular fill; and 2) close reinforcement spacing. Bulging at the face or problems concerning the internal stability of a GRS mass indicates that either one or both of the two rules were not followed. Building a GRS mass is easy; a row of blocks, a layer of compacted fill to the height of the blocks 0.2m (8 inches), followed by a layer of geotextile that extends between the layers of block. The 1-2-3 process is repeated until the wall height is reached.

Typically, the primary reinforcement is spaced at 0.4m (16 inches) for walls and 0.2m (8 inches) for load bearing, abutment applications. The spacing of reinforcement in the zone directly beneath a bearing area can also be reduced to less than 0.2 m to provide additional confinement and mass stiffness. In recent years, the type of reinforcement most often used in GRS is polypropylene geotextile; this is because of its affordable price and suitable material properties. Another advantage of this type of geotextile is that it can be rolled out parallel to the face of the wall, as opposed to perpendicular, thus optimizing material quantities.

The requirements for the connection of the modular block to the GRS were based on the fact that GRS can effectively restrain lateral deformation of the soil without the requirement for mechanical connection. As previously stated, the recommended primary reinforcement spacing for walls and abutments is 0.2m and 0.4m, respectively. For the case of walls where the primary reinforcement layers are spaced at 0.4m, it is recommended that secondary layers or tails be placed between the primary layers to allow for better soil compaction at the face of the wall. The secondary reinforcement layers connect the facing block to the reinforced soil mass and reduce lateral loads against the modular block face (Figure 1, Figure 5).

As illustrated in Figure 1, the thrust against each course of the block can be described in terms of bin pressure as a function of the reinforcement spacing and not wall height (Wu J.T. 2007[8]). A practical implication of this is that the facing block should only resist lateral movement such that an active condition can develop within the wedge soil confined between the layers of the reinforcement. For the case of a GRS mass consisting of reinforcement spacing of 0.2m (8”) and a soil, that has a unit weight of 125 lb/ft^3 and friction angle of 34 degrees, the lateral thrust produced is about 11 lbs/ft. The weight of the split face CMU is more than double this thrust value developed from the small wedge of soil at the face behind the facing block.



$$\sigma_h = \gamma\, S\, K_a = \gamma\, S \tan^2(45° - 34°/2) = 125\, S\, (0.283) = 35.4\, S$$

$$\begin{aligned} F &= 2/3\, \sigma_h (7/10\, S) + 5/6\, \sigma_h (3/10\, S) \\ &= 43/60\, \sigma_h\, S \\ &= 25.4\, S^2 \text{ lbs/ft} \end{aligned}$$

Figure 1. Thrust on Facing Elements (assuming yielding face)

The important issue illustrated in Figure 1 is that the frictional connection between the facing blocks and reinforcement is sufficient in resisting lateral earth pressures without the requirement for a mechanical connection system. This simple method of connection is designed to allow the blocks to yield slightly against thrust to reduce pressures against the face. The lateral pressure within a GRS structure is a function of reinforcement spacing, and not wall height. The degree of frictional connection between the facing block and reinforcement is a function of the normal force due to the weight of the facing blocks and can vary between several hundred to more than a thousand pounds. Even though the modular block on a GRS wall provides some secondary confinement, the GRS structure is internally supported and the primary function of the block is a façade and form for each lift of fill during construction.

A GRS mass is a composite material with a distinctive stress-strain relationship depending on the type of reinforcement and backfill material. It has also been observed that both the reinforcement and the soil strain together act as a composite GRS material when subjected to vertical stress. The strain compatibility of the soil and reinforcement has been observed to lateral strains of more than 2%. Creep rates of GRS mass constructed with a reinforcement spacing of 0.2m will typically accelerate at about 2.5% lateral strain. A GRS abutment will have a reinforcement strain less than 0.5% at an allowable bearing stress of 200 kPa, (Adams M.T. et al. 1999 [9], Adams M.T., 1997).

THE GRS INTEGRATED BRIDGE SYSTEM.

In the past, GRS technology had been used to support the bridge directly on the abutment wall (Abu-Hejleh, N. et. el., 2001 [10], J.T.H. Wu et. el., 2001 [11], Devins et. el., 2001 [12], Saunders, S. A. et. el., 2007 [13]). There are also numerous undocumented applications of this technique in the private sector. In these cases, the beams were supported on an intermediate concrete foundation, or sill perched on top of the abutment wall.

The method is similar to the placement of traditional spread footings on an approach fill embankment. Some obvious differences are that the allowable bearing stress on GRS is greater than the stress allowed on compacted fill and the span length for the bridge supported on GRS is shorter because the wall can be constructed vertical, where as an approach embankment typically has a 2:1 fill slope.

Additionally, the more traditional method would include a stem wall cast into the back of the spread footing to retain the approach fill behind the beam. The approach detail also frequently includes an approach slab to smooth the transition for the approach onto the bridge (DiMillio A. F., 1982 [14]).

In the “GRS Integrated Bridge System,” the adjacent concrete box beams are supported directly on the GRS abutments, without a concrete footing or elastomeric pads. The bridge has no cast-in-place concrete or approach slab. Figure 4 illustrates a typical cross section of a GRS integrated abutment to show that GRS is compacted directly behind the bridge beams to form the approach way and to create a smooth transition from the roadway to the bridge.

The reinforcement layers behind the beam-ends are wrapped to confine the compacted approach fill against the beam-ends and the adjacent side slopes to prevent lateral spreading. Since the wrapped faced GRS fill behind the beam-ends is free standing, the active lateral pressure against the beam-ends is considered negligible as indicated in Figure 1. The wrapped face fill also prevents migration of fill during thermal bridge cycles and vehicle live loads.

Similarly, the placement of GRS behind the stem wall on a traditional bridge abutment would reduce lateral pressure and the rotational force against the wall. This detail would also confine the fill and prevent the loss of the material that can create a bump (White, D., 2005 [15]).

In the integrated system, GRS is also compacted against the beam sides on the wing walls. Guardrail posts are installed with a 1.1m (3.5ft) setback distance from the face of the GRS wall. Asphalt pavement is placed on the bridge and approach without a conventional joint system at the bridge ends. A layer of paving fabric is extended from the concrete box beams onto the approach fill to bridge the transition. The integrated bridge system is designed for the bridge and the adjacent road to settle together providing a bump free, smooth ride for drivers on and off the bridge. The system is also expected to reduce maintenance requirements.

This type of abutment is not suitable for every bridge building assignment; however, the technology is well suited for single span bridges of less than 36.6m (120 feet), but not for water crossings where the potential for scour is high.

DESIGN BACKGROUND

The Bowman Road Bridge, in Defiance County, Ohio, was the first production bridge to use GRS technology to integrate the bridge substructure and superstructure with a jointless pavement. State of the practice in this region is to build typical bridges with concrete box beams on pile cap abutments with 2:1 side slopes. As shown in Figure 2, placement of the Bowman Road Bridge on GRS abutments reduced the beam length of 40.8m (134 ft) to 25m (82 feet).

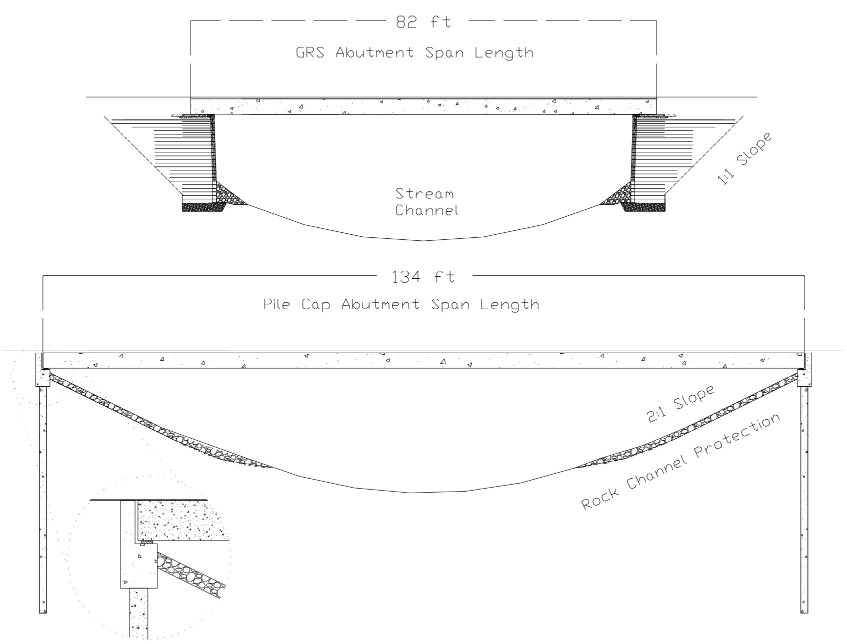


Figure 2. Illustration of Bridge Span Length for GRS and Pile Cap Abutments

The center of bearing was 0.6m (2 ft) behind the back of the concrete modular facing block. The width of the bridge seat was 0.91m (3 ft). The setback distance or gap between the back of facing block and bridge seat was 0.15m (0.5 ft).

Typical concrete box beams were used to form the 10.4 m (34 ft) wide superstructure. The weight of the beams and asphalt produced an equivalent bearing stress of 163 kPa (1.7 tsf). The setback distance was added to the width of the bridge seat for an effective bearing width of 1.1m (3.5ft) to calculate the bearing stress.

The economics of this method is apparent when comparing the cost of the main components of these two bridge systems. As indicated in the following table, the cost of building the GRS abutments more than offset the cost for the longer beams and installation of the deep foundation. The price estimate to build the bridge on pile cap abutments was based on a bid provided by a contactor. The actual cost to construct the bridge supported on GRS abutments was provided by Defiance County. The cost savings was nearly 25 percent.

Abutment Type Cost Comparison	
GRS Abutment $95,000	Conventional cap Abutment on piles $105,000
Beams and Waterproofing $171,000 (34'x 82')	Beams and Waterproofing $233,000 (34'x 110')
Total $266,000	Total $338,000

Table 1. Simplified Bridge Cost Comparison: GRS vs. Pile Cap Abutments

Another advantage is the bridge was constructed in less time. The Bowman Road Bridge was built in about 6 weeks versus a construction time of several months if the bridge was supported in a conventional abutment. The county has since streamlined the GRS construction process and is now capable of completing a bridge in about 2 weeks, with a cost savings of nearly 30 percent depending on the span length and abutment height. The time to build these bridge systems can be reduced even more by simultaneously building both abutments with two separate labor crews. The typical labor crew consisted of four laborers and one equipment operator.

During the past two construction seasons, Defiance County has switched from installation of stubby pile cap abutment to the integrated GRS abutments because of cost and flexibility of construction. In addition, the county has also elected to use GRS abutments instead of box culverts.

The Bowman Road Bridge in Defiance County was built in the fall of 2005 with design and construction guidance from the FHWA. Design of the abutments was based on the first author's knowledge of GRS for load bearing applications in combination with recommendations provided in NCHRP Report 566.

The computer program MSEW was also used to check for external global stability and reinforcement strength against the development of an active failure wedge. The MSE design requirements for eccentricity, reinforcement pullout and connection were not applied because MSE design methodology does not account for geosynthetic soil interaction.

Another deviation from MSE design policy is the selection of the reinforcement strength reduction factor. The allowable reinforcement strength was set at 0.3 the reinforcement's ultimate strength, instead of the recommended reduction factor of 0.03 for polypropylene reinforcements as outlined in the FHWA design guidance for MSE walls (Elias, V. et. el, 2001 [16]). The reinforcement spacing in a GRS wall is close, is internally stable and does not creep when properly constructed. For this reason, the reduction factor for creep is essentially omitted in the calculation of the allowable strength from the reinforcement ultimate strength. A combined reduction factor of 3.33 is applied to guard against installation damage and durability.

A GRS structure is internally supported and can be efficiently built with a lightweight simple CMU block that is frictionally connected to the geotextile reinforcement layers. GRS structures may also be built with MSE blocks. Proprietary MSE systems are built with a distinctive MSE block-reinforcement combination. Each unique MSE segmental wall system has a connection method that specifies special pins, keys or block shapes that mechanically connect geosynthetic reinforcement to a particular block. Incorporated into the design of these systems (many of which are proprietary) is a technique to resist shear between each course of block.

A simple standard split-faced CMU is representative of the products available for modular block wall construction are an economical alternative to a MSE block. The manufacturing process and mix design for split-faced CMU block is often identical to segmental retaining wall (SRW) block. Both are dry-cast concrete products that can be made to a particular performance specification depending on geographic location or application (Chan, C., et. el., 2007 [18]). A standard CMU is 8 inch deep, whereas SRW units are most commonly 12-inches deep. Other than size and weight of the block, one difference between a CMU and SRW block is the allowable height tolerance across the length of the block, 3 mm and 1.5 mm, respectively.

Figure 3 shows each abutment face for the Bowman Road Bridge. The Bridge was skewed approximately 24.5 degrees and super elevated about 0.88m (2.9ft).

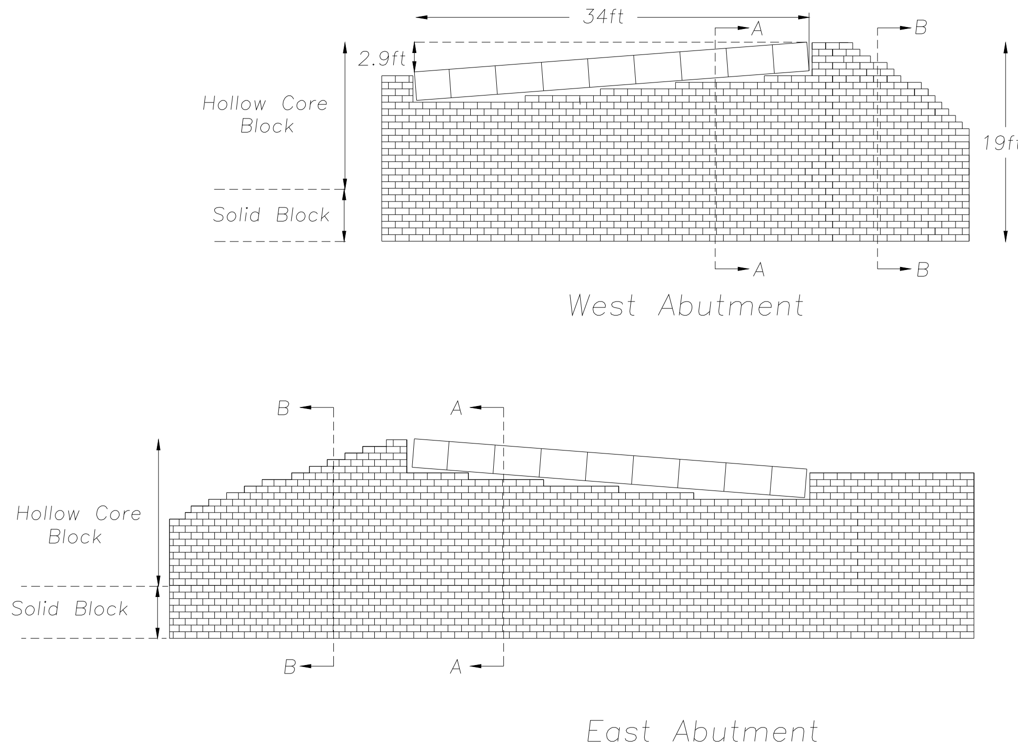


Figure 3. Face View of GRS Abutments to Show Bridge Super Elevation

GRS ABUTMENT CONSTRUCTION

The fill material used to build the abutments was 10 mm (3/8 inch) diameter crushed limestone aggregate, classified between a fine gravel and coarse sand. This material was easy to spread and compact. Direct shear box tests on the fine gravel indicated that the friction angle was 37 degrees. Two types of woven polypropylene geotextiles were used to build the GRS abutments; the wide width strength of the fabrics was a 70 kN/m (4800 lb/ft) and 30 kN/m, (2100 lb/ft) strength material.

Figures 4 and 5 show the cross sections for the abutment and wing walls, respectively. As explained in the previous section, the primary reinforcement spacing for GRS wall applications is 0.4m (16 inches). As illustrated in Figure 5, the reinforcement schedule for wing wall is 0.4m. Figure 5 also shows secondary reinforcement layers or tails between the primary reinforcement layers. The short length of the reinforcement tail is typically about 1 m (3.3 ft) and their purpose is to allow proper compaction at the face of the wall behind the block. The tails also ensure that the facing block is connected adequately to the GRS mass. A tail roll can be quickly made by cutting a 1 m section from a roll of geotextile with a chainsaw.

It is important to note that the GRS wing walls shown in Figure 5 could have been built with the 30kN/m strength geotextile. However, since 70kN/m strength was used to build the abutment (figure 4) it was necessary to also use the same reinforcement on the wing walls to make each row of block level. The thickness of the each reinforcement type was different.

The Bowman Road Bridge spans Powell Creek that frequently floods but does not produce sufficient water velocity to cause scour. The bridge was still protected against scour by building it on a reinforced soil foundation (RSF) in a geotextile. The first course of the concrete modular block was placed directly on a sheet of geotextile that encapsulated the RSF.

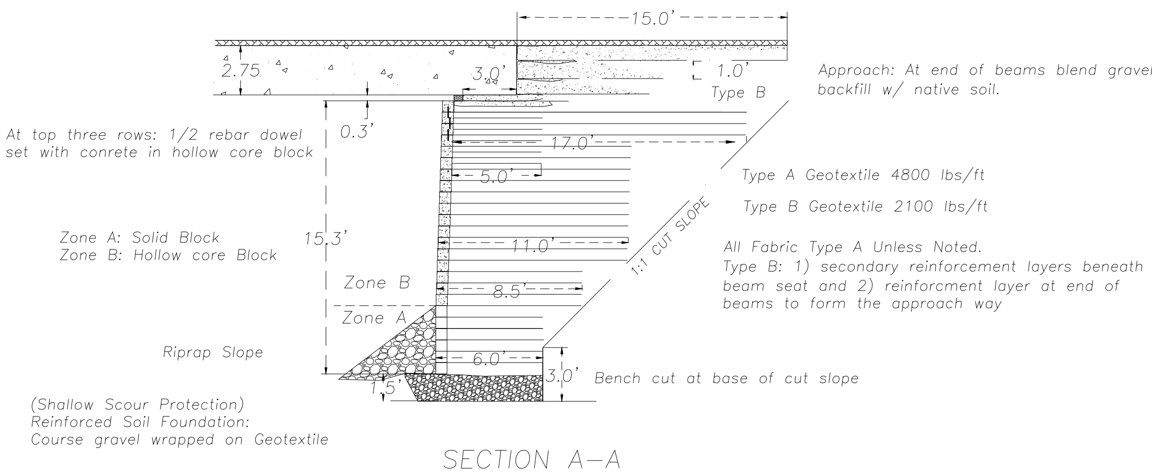


Figure 4. Reinforcement Schedule (Abutment and Approach Way)

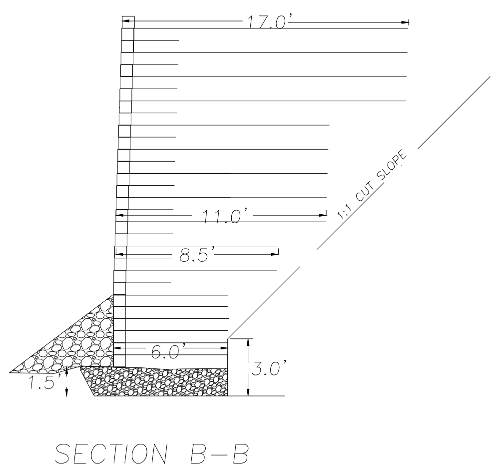


Figure 5. Reinforcement Schedule (Wing Walls)

The sheets of geotextile fabric were used to encapsulate the RSF. They were overlapped so that the top piece was upstream similar to the way roof shingles shed water. The depth of the RSF was about 0.46m (1.5ft). It was built with two layers of compacted, well-graded gravel layered with geotextile in the middle.

The concrete modular blocks used to face the abutment were split-faced concrete masonry units (CMUs). The blocks were frictionally connected to the GRS mass with a layer of reinforcement between the rows of modular blocks. The height of each abutment wall was about 4.6m (15 ft). The walls were built with two different colors and types of CMU block; a red solid core block in the lower base section of the wall and a grey hollow core block in the upper 2/3 section. A riprap talus was built against the red block to armor the face of the wall against scour. Exposure of red block during inspection is a visual indication that erosion or scour may have occurred.

The abutment and wing walls were built as illustrated in Figures 4 and 5 following the two basic rules and the 1-2-3 process as explained in the preceding section.

GRS INTEGRATION

As shown in Figure 4, at the top of the wall, the spacing of the reinforcement was reduced in the area beneath the bridge beams to significantly increase the strength of the GRS composite. This detail allows load to be placed directly behind the face with insignificant lateral deformation. On top of the wall, a 0.1m (4 inch) thick bridge seat was constructed with a layer of fine gravel encapsulated in a sheet of the 70 kN/m geotextile reinforcement.

To form the Integrated Abutment the box beams were placed directly on the GRS abutments. The bridge was built without cast-in-place concrete. The bridge does not have an approach slab; instead, GRS was compacted directly behind the bridge beams to form the approach way and create the smooth transition from the roadway to the bridge as illustrated in Figure 4.

As explained above, the GRS was compacted against each beam end and was layered to create a gradual transition from the abutment to the road. The layers behind the beam-ends were wrapped to confine the compacted approach fill against the beam-ends and the adjacent side slopes to prevent lateral spreading. Since the wrapped faced GRS fill behind the beam ends is free standing, the active lateral pressure against the beam ends is considered to be negligible. The wrapped face fill also prevents migration of fill during thermal bridge cycles and vehicle live loads. Pressure cells were installed at the beam ends to measure the passive earth pressure due to thermal expansion and contraction cycles.

PERFORMANCE MONITORING

The bridge and abutments were instrumented by FHWA to evaluate performance and GRS-bridge interaction. A survey station was installed to record bridge settlement and movement of the GRS abutments. Earth pressure cells were installed to measure stress beneath the beams and at the base of the GRS abutments. Several earth pressure cells were also installed behind the beams to measure passive earth pressures against the approach fill due to the thermal cycles of the beams. Strain gauges were installed in the beams during casting to correlate thermal expansion and contraction cycles to the pressure at the beam ends.

The instrumentation and settlement will be monitored for a 2 to 3 year period. Based on observations and collected data to date, the bridge is performing very well. There has been very little movement or settling and no pavement cracking at the approach. The stress due to thermal cycles of the beam ends against the approach fill is in the range of about 25kPa, (3.6psi).

Figure 6 is a graph that shows the settlement of each abutment with time. Included in each graph is the settlement of both the abutment wall and bridge beams. The weight to bridge on the GRS abutments is equivalent to bearing stress of 163 kPa (1.7 tsf). The graph indicates that the average total settlement at the end of the beams is about 0.87 inch (22 mm). The settlement of the bridge is the result of deformation within the GRS mass and consolidation of the foundation soil beneath the abutments. As indicated in the graph, consolidation of the foundation soil is equal to the settlement of the abutment walls, 0.45 inch (13.7mm). The average settlement difference between the beam and the face of each GRS abutment wall is the result of deformation within the GRS mass caused by the surcharge weight of the superstructure is 0.33 inch (8.4mm).

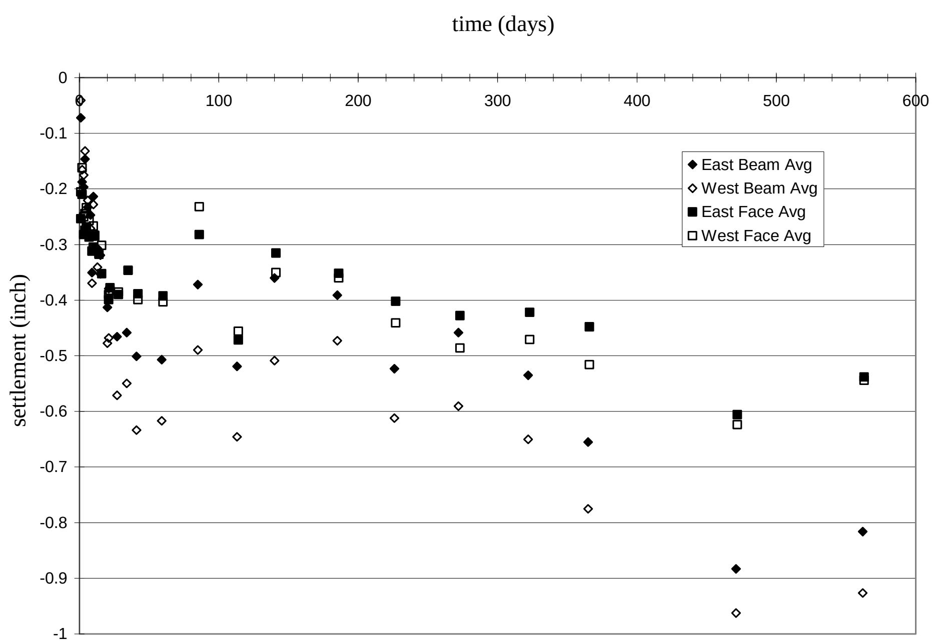


Figure 6 Bowman Road Bridge (Post Construction Settlement)

The differential settlement between each end of the bridge, .11 inch (2.7 mm), is negligible. The pavement overlay on this joint-less bridge system has not cracked to date.

Defiance County has built 10 additional bridges supported on GRS abutments. All are performing excellent.

Figure 7. Construction of GRS Abutment

Figure 8. Placement of Concrete Box Beam on GRS Abutment

Figure 9. Placement of Concrete Box Beams Directly on the GRS Abutment
(note absence of concrete footing)

Figure 10. Placement of Beams on GRS Abutment for the Glenberg Bridge

Figure 11. Integration of Approach Way for the Glenberg Bridge – Next Day After Beam Placement

Figure 12. Completed Glenberg Bridge Above Flood Waters.
(note: wood guard rail posts should never be used in GRS applications)

Figure 13. Completed Bowman Road Bridge

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